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Summary Report

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SUMMARY REPORT

THE EFFECT OF GEOGRID REINFORCEMENT ON THE STABILITY OF EMBANKMENTS ON A SOFT SENSITIVE CHAMPLAIN CLAY DEPOSIT

Prepared for

TRANSPORTATION DEVELOPMENT CENTRE

by

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17. Abstract

Three instrumented test embankments were constructed to failure at a site in St. Alban, Quebec, in order to study the effect of geogrid reinforcement on the stability of embankments on a soft sensitive clay deposit. The geogrid, Tensar SR-2, was incorporated in the base of two of the three embankments using different configurations. One of the embankments was reinforced with two horizontal layers (strips) of geogrid, while the other was reinforced with an equal quantity of geogrid, constructed to form a sand filled cellular structure. The interpretation of monitoring results provides a comparison of the foundation response, and the performance of the geogrid up to and immediately prior to failure. The forces mobilized at failure were examined by back analyses of the failed embankments using limit equilibrium methods. These analyses indicated that the reinforcement caused improvements in failure heights of about 34 and 17.5 per cent in the strip and geocell reinforced embankments, respectively. The significant effect of the crust strength in the prediction of failure heights of the test embankment is identified in the study. It is further concluded that there is no direct evidence to show that the geogrid has a beneficial effect on the redistribution of stresses in the foundation, nor in retarding the propagation of local failed zones prior to failure. Its apparent effect is to maintain the integrity of the fill and allow more extensive propagation of failed zones, before a general failure state is achieved in the foundation.

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17. Résumé

Trois remblais expérimentaux instrumentés ont été élevés jusqu'à rupture, à St-Alban (Québec), afin de connaître l'effet d'une armature en géotextile sur lar stabilité des remblais établis sur sol mou argileux. La géogrille utilisée, de marque Tensar SR-2, a été mise en place à la base de deux des trois remblais, selon deux configurations différentes. Le premier a reçu deux nappes horizontales de géogrille tandis que dans le deuxième, une même quantité de géogrille a servi à la réalisation d'une armature alvéolaire remplie de sable. L'interprétation des résultats des mesures a permis une appréciation comparative du comportement des assises et de l'action des armatures jusqu'à la rupture des remblais et immédiatement avant qu'elle ne survienne. Les forces en jeu à la rupture ont été analysées à posteriori par la méthode des états limites. On a pu constater que l'armature en nappe et l'armature alvéolaire ont permis d'augmenter la hauteur de rupture des remblais par environ 34 et 17,5%, respectivement. L'étude a mis en évidence l'importance de prendre en compte la résistance de la croûte d'argile dans la prédiction des hauteurs de rupture. Par contre, elle n'a pu trouver de preuve directe indiquant que la géogrille aurait une influence positive par la redistribution des contraintes dans l'assise des remblais, et qu'il retarderait la propagation des ruptures localisées jusqu'à la rupture finale. Il semble qu'il ait pour effet de retarder l'échéance de la rupture généralisée tout en autorisant une propagation plus étendue des zones de rupture localisées.

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1.0 INTRODUCTION

The construction of embankments on soft sensitive clays is affected by the brittle, strain softening behaviour of these deposits. Additional construction costs associated with the need for stage loading, height and slope restrictions probably amounts annually to several million dollars across Canada. The magnitude of this problem has necessitated the development of practical engineering solutions, including the improvement of stability of embankments by means of geogrid reinforcing inclusions.

Review of the reinforcing action of artificial inclusions in earth fills has suggested that it is probable that the inclusion of reinforcement in embankments is a cost effective means of improving embankment performance on soft soils. It could be considered that the inclusions will produce, within the base of the embankment, an artificial crust with well defined properties. While crustal thickness and rigidity are important variables affecting embankment performance, the variation and subsequent determination of these parameters cannot be rationally accounted for in design at the present state of knowledge.

Despite exhaustive research in the past two decades on the prediction of embankment stability on soft clays, a commonly accepted method of analysis is still not available to the design engineer. Therefore, a design philosophy for reinforced embankments on soft clays is subjected to the same uncertainty, and a satisfactory method must be based on a fundamental understanding of the effect of the artificial inclusion on the foundation.

The purpose of this study is to examine the effect of geogrid reinforcement on the stability of embankments on soft sensitive clay. To this objective three test embankments were constructed to failure at a site in St. Alban, Quebéc. Two of these embankments were reinforced using geogrids in different configurations. The behaviour of the embankments, geogrid and foundations were monitored with in situ instrumentation. From the interpretation of the data collected in the three embankments, conclusions pertaining to rational design of reinforced embankments on soft sensitive clays, have been drawn.

This summary report on the research project is jointly prepared by Golder Associates and Université Laval. The report describes the preconstruction design activities, the field work and the monitoring results of the instrumentation. Conclusions and recommendations drawn from interpretation of these data are subsequently summarized.

2.0 SPECULATED EFFECTS OF GEOGRID REINFORCEMENT

A review of published literature (Milligan and La Rochelle 1984, Trak et al 1980, Tavenas and Leroueil 1980, Ladd and Foott 1974) revealed a number of uncertainties associated with conventional limit equilibrium analyses of embankment stability on soft clays. In particular, the assumptions often made with respect to the strength of the foundation and the failure mode may not represent actual field behaviour.

It has been postulated (Milligan and La Rochelle 1984) that the effect of artificial inclusions at the base of embankments could be to reduce the magnitude of shear stresses at the fill ground interface, by restricting the spreading action of the

fill. This change in external boundary stresses could result in a beneficial redistribution of shear stresses in the foundation. Based on this hypothesis, it can be demonstrated by mathematical modelling that a stiff reinforcement on a foundation with a thin crust reduces the shear stresses at shallow depths. This is analogous to the action of an additional crust, which tends to move the zone of maximum shear stresses to deeper, more competent materials resulting in a more stable embankment for a given surcharge.

3.0 PRECONSTRUCTION ACTIVITIES

3.1 SITE AND GEOGRID SELECTION

The behaviour of soft sensitive clays under embankment loading has been the subject of research carried out by the geotechnical group at Université Laval, Quebéc over the last fifteen years. The main focus of this work was the construction of four test embankments at St. Alban, Quebéc in 1972. The monitoring records and analyses of these embankments, together with laboratory investigations, have provided reliable data on the performance of the St. Alban clay. The site of these test embankments (Figure 1) is therefore an ideal location to construct instrumented test embankments to study the effect of geogrid reinforcement on the stability of embankments on soft, sensitive clays.

A suitable artificial inclusion in the embankment fill would be fairly rigid, environmentally inert and have well defined physical properties. The most appropriate material available at the time of the study was a high strength polyethylene geogrid, Tensar SR-2. The quality control tensile strength of the material at high strain rates is 79 kN/m width. Typical mechanical properties of the geogrid as provided by the manufacturer are shown in Figure 2.

3.2 SITE CHARACTERIZATION

The subsurface conditions across the general site area were inferred from a detailed geotechnical investigation carried out in 1972 by Université Laval. This indicated that the site is generally underlain by a 1.5 m thick weathered clay crust, which is in turn underlain by an 8 m thick deposit of soft grey to blue marine clay. Studies on this soft clay indicated that the deposit is of low to medium plasticity, with measured water contents appreciably higher than the liquid limit of the material. The undrained shear strength of the clay, as inferred from field vane tests, increases linearly from a minimum value of about 10 kPa under the crust to about 33 kPa at 9.1 m depth, with a maximum variation in strength of about 20 per cent. The measured vane strength of the crust was very variable, but averaged about 38 kPa. The undrained shear strength of the material at large strains (USALS) (La Rochelle et al 1974) was also investigated by laboratory testing on large diameter samples, and was noted to equal about 90 per cent of the vane strengths below the crust. A typical profile of the geotechnical properties, reproduced from Trak et al (1980), is given in Figure 3.

A geotechnical investigation was carried out in May and June, 1984, as part of this research project, to confirm that subsurface conditions in the intended area of construction are consistent with those in the 1972 study. Based on the results of this investigation, the three proposed test embankments were located as shown in Figure 4.

3.3 EMBANKMENT DESIGN

The design of the proposed test embankments was based on the results of the 1972 test embankment (La Rochelle et al, 1974), using the USALS strength profile. The 1972 failed embankment was also used to calibrate computer programs of slope stability analysis developed by Université Laval (STAB) and Golder Associates (SARMA).

A stability program based on limit equilibrium and which allows the incorporation of a horizontal stabilizing force to simulate the reinforcing action of the geogrid was also developed (BTENSAR) and calibrated against the other two programs (STAB and SARMA). A failure height of 5.2 m was calculated for an embankment reinforced with two horizontal strips of Tensar SR-2 geogrid. Stablizing berms were constructed at the back and sides of the embankments to encourage failure on the preferred side, where the instrumentation was concentrated.

Two different configurations of the geogrid were devised to examine the effect of rigidity of the reinforcement on embankment behaviour. Two horizontal layers of Tensar SR-2 were incorporated at the base of Embankment No. 2 (the strip reinforced section). A geocell mattress, using the same quantity of material, was designed for Embankment No. 3. The geocell consisted of interconnected triangular cells of geogrid, 1 m high, which were subsequently filled with sand (Figure 5).

A finite element analysis of the reinforced embankment adopting a hyperbolic stress-strain model for the soil was carried out. This indicated that the reinforcement would

have an effect in restraining the propagation of failure in the subsoil, and reduce both the excess pore pressure generated and the magnitude of deformation under the embankment.

3.4 INSTRUMENTATION

A variety of geotechnical instruments were utilized to monitor the behaviour of the test embankments. The layout of the instruments was similar in all three embankments so that direct comparisons could be made. A typical layout is shown in Figure 6. The type and quantity of instrumentation installed is listed in Table 1.

	Quantities of Instrumentation		
Type of Instruments	Control (No. 1)	Strip Reinforced (No.2)	Geocell Reinforced (No.3)
Pneumatic piezometers	14	16	16
Hydraulic piezometers	10	5	14
Electrical piezometers	1	2	4
Settlement plates	11	11	18
Monuments (settlement)	14	14	14
Vertical extensometers	4	3	3
Inclinometers	2	2	2
Horizontal extensometers	2	2	2
Monuments (horizontal displacement)	14	14	14
Total pressure cells	4	5	4
Tensar load cells	-	2	2
Bison strain gauges	-	16	10
Differential extensometers	-	2	-

Table 1 Quantities of Installed Instrumentation

a) PLACEMENT OF GEOGRID REINFORCEMENT

b) GEOGRID INSTRUMENTATION

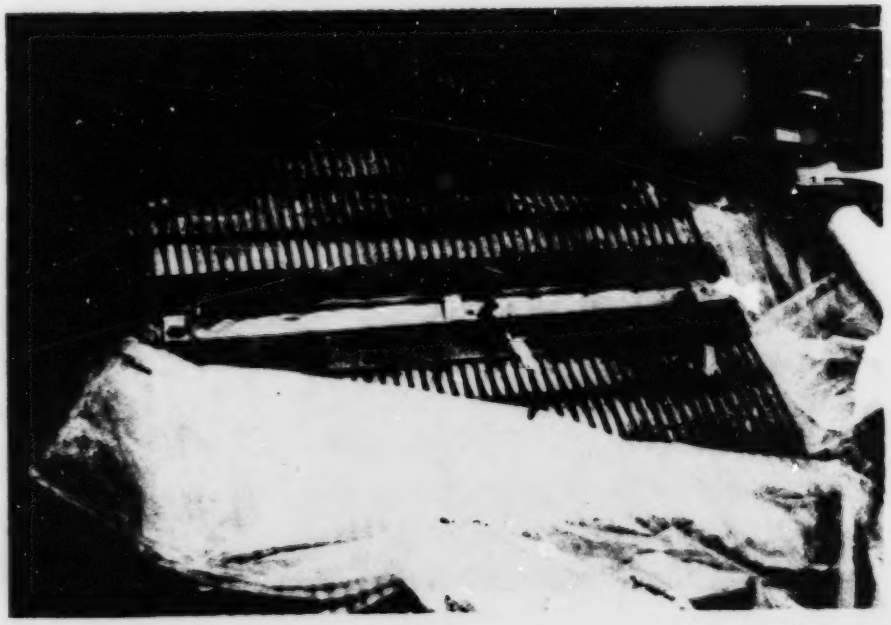

PLATE I PLACEMENT AND INSTRUMENTATION OF STRIP REINFORCEMENT

a) STRIP REINFORCED EMBANKMENT (No. 2)

b) GEOCELL REINFORCED EMBANKMENT (No. 3)

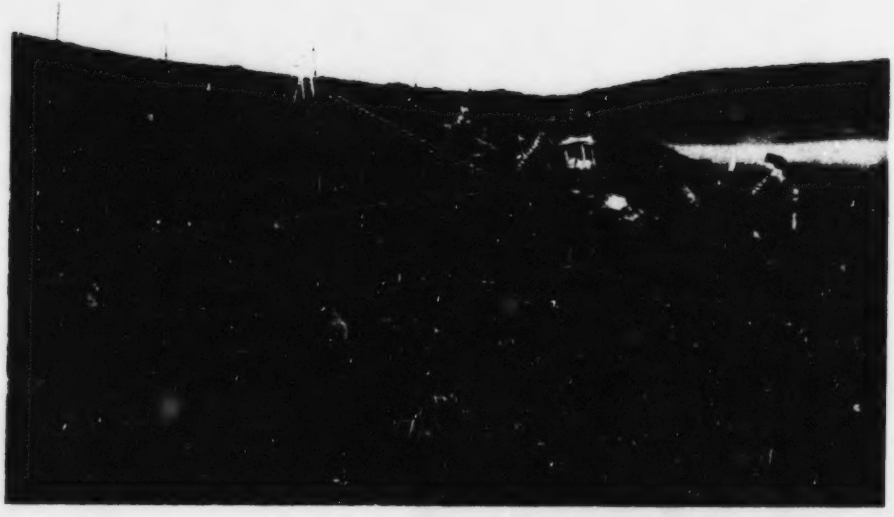

PLATE 2 EMBANKMENT FAILURES

4.0 CONSTRUCTION

Installation of subsurface instrumentation was completed on July 9, 1984. After the instruments had stabilized, the construction of the strip reinforced embankment (No.2) commenced on August 13, 1984. The instruments were read at least twice a day, and the results were continuously updated and reviewed using a computerized data base. The embankment side stopes were maintained at a gradient of 1.5 horizontal to 1 vertical, and two 0.3 m thick lifts of fill were generally placed each day. Failure of the embankment occurred on August 23, 1984, at a height of about 6.0 m above original ground surface. The instruments were regularly monitored during the subsequent week, after which period the nature and location of the failure surface was studied by a local excavation and piezocone testing. Some instruments were also recovered for use in the second and third embankments.

Subsurface instrumentation for the control embankment (No.1) was completed on August 20, 1984. After three weeks of stabilization, the construction of the control section commenced on September 10, 1984. The fill used for construction of the first (strip reinforced) embankment was reused in building the second (control) embankment. The rate of construction of the control embankment was the same as that of the first one. The embankment failed on September 19, 1984 at a height of about 5.1 m above original ground surface. Two failures were observed; in the anticipated direction of failure and on the side of the stabilizing berm where a trench had been excavated during excavation of the fill which formed the first embankment. The two failures occurred simultaneously.

Subsurface instrumentation for the geocell reinforced embankment was completed on September 18, 1984. Assembly of the geocells commenced on September 24, 1984 and was completed on

October 3, 1984. In order to avoid preloading the foundation and to maintain a consistent rate of embankment loading the geocells were left unfilled until the start of embankment construction.

Construction of the third embankment commenced on October 9, 1984. The geocell mattress was filled in about 3 days and the embankment constructed from October 9 to October 17. The embankment failed on the night of October 17, 1984, at a height of about 4.5 m above original ground surface. The surviving instruments were regularly monitored for a week, during which period some additional field vane and piezocone testing were also carried out.

The density of the placed fill was measured by means of a portable nuclear density apparatus. An average of 10 to 12 tests were carried out per day. The results of the density tests on the three embankments are summarized in Table 2.

	Control (No.1)	Strip Reinforced (No.2)	Geocell Reinforced (No.3)
No. of Tests	78	119	118
Average Total Unit Weight (kN/m^3)	18.1	16.9	18.2
Average water Content (percent)	6.3	5.4	9.1
Average Dry Unit Weight (kN/m^3)	17.0	16.0	16.7

Table 2 Results of Density Tests

5.0 FOUNDATION RESPONSE

5.1 SURCHARGE

To maintain an approximately constant rate of loading on the foundation, the height of the test embankments was increased at the same rate. However, the difference in water contents of the fill caused a variation of surcharge pressure of about 5 per cent. Therefore, the surcharge pressure of the fill at the centre of the embankment has been taken as a basis for the comparison of embankment behaviour. The net settlement of the embankment was considered in the calculation of this surcharge pressure.

5.2 PORE PRESSURES

The behaviour of the piezometer showing the maximum response, PN-15 (Figure 7) is used to illustrate the build up of pore pressures under the centre of the embankment. It can be seen that the pore pressure response at the centreline is very similar in all three embankments. The pore pressure coefficient $\bar{B}$, is given by the ratio of excess pore pressure (Δu) and the increase in vertical pressure ($\Delta\sigma v$) which equals the increase in the maximum principal stress ($\Delta\sigma_1$) under the centreline at the original ground surface, times an elastic stress influence factor (I).

A $\bar{B}$ value of about 0.5 was observed for an increase in vertical stresses up to between 37 and 41 kPa. Up to this stress level a consolidation process occurred during construction. At higher stresses $\bar{B}$ became constant at a value close to unity and it can be inferred that the preconsolidation range of the clay at this location (PN-15) is in the range of 18 to 20 kPa (Figure 7). At a vertical stress increase in excess of about 60 kPa, the slope of the pore

pressure response line further increased to about $\bar{B}$ = 1.3, and remained virtually constant until failure occurred. The increase in $\bar{B}$ above unity indicates local failure of the clay. At the location of PN-15 this occurred, at a total vertical stress increase of about 60 kPa.

The excess pore pressure contours under the axis of the embankments were also compared to examine the extent of failure zones in the foundation. It was noted that the excess pore pressure contours under all three embankments at the same surcharge are similar with slightly higher pore pressures under the toe of the reinforced embankments. (Figure 8).

No definite evidence that the geogrid reinforcement is effective in reducing the pore pressure response in the foundation was obtained.

5.3 VERTICAL DEFORMATION

The settlements under the centreline of the embankments show the same trends, with a gradual increase in the rate of settlement with respect to surcharge loading (Figure 9). The centreline settlements of all three embankments were in the narrow range of 220 mm to 235 mm just prior to failure.

5.4 HORIZONTAL DEFORMATION

The shape of the deformed inclinometer casings and the trend of deformation are similar in all of the embankments, except that at high surcharges the deformations below the weathered crust (at 3 m and 5.5 m depth) are significantly greater in the reinforced embankments (Figure 10). The rate of horizontal deformation below the weathered crust increased quite dramatically when the centreline surcharge exceeded about 74 kPa under the geocell reinforced embankment and about 78 kPa under the strip reinforced embankment.

The comparison of horizontal deformations under the embankments provides no evidence that the reinforcement had a beneficial effect on the redistribution of shear stresses in the subsoils, nor did it help to retard the propagation of failed zones in the foundation. It would appear that the effect of the geogrid is to maintain the integrity of the embankment fill while deformations in the foundation develop to a greater extent than would have occurred at failure under an unreinforced embankment.

6.0 PERFORMANCE OF THE REINFORCEMENT

6.1 LIMIT EQUILIBRIUM ANALYSES

Limit equilibrium analyses were carried out to determine the effect of the reinforcement on the failure heights, and to calculate the forces mobilized in the reinforcement at failure. The data relevant to the analyses is summarized in

EMBANKMENT	Height[1] at Failure (h_c) (m)	Depth[2] of Failure (m)	Fill Parameters		
			Unit wt.(γ) (kN/m^3)	Surcharge (h_c) (kPa)	ϕ' (degrees)
1972 Unreinforced	4.0	4.0	18.9	75.6	44
1984 Unreinforced	5.3	4-5.5	18.1	95.9	34
1984 Strip Reinforced	6.2	4-5.5	16.9	104.8	34
1984 Geocell Reinforced	4.7	3-6	18.2	85.5	34

Table 3 Details of Failed Embankments

(1) The height at failure is taken as the height above the original ground surface plus the average of the settlement of the base under the centreline and the shoulder of the embankment, and, minus the heave of the toe.

(2) Depth of failure from piezocones after failure and inclinometer readings during construction.

Table 3. Consistent with field observations of the actual failures, circular arc failure surfaces were assumed through out the analyses.

To compare the failed embankments a consistent assumption of the shear strength profile of the subsoil has to be devised. It is generally recognized that the available shear strength at failure for near normally consolidated clays below the crust is related to their preconsolidation pressure. Trak et al (1980) have also shown that at the St. Alban site this is represented by the USALS profile which is equal to 0.22 times the preconsolidation pressure. The mobilized shear strength at the site is about 90 per cent of the vane strength and this factor was therefore applied to the measured vane strengths in order to obtain the mobilized shear strength below the weathered crust.

A more difficult problem is the appropriate strength of the crust to be adopted in the analyses. Back analyses of the 1972 failure found the best agreement, using vane strengths, when the "mid-depth strength" was assumed for the crust (La Rochelle et al 1974). Adopting this method for determining the shear strength of the crust and factoring the vane strength profiles by 0.9 below the crust as discussed above, factors of safety of 0.99 and 1.00 were obtained for the 1972 and 1984 unreinforced embankments, respectively, using the BTENSAR program. The effect of varying the friction angle of the fill was shown to be insignificant for the range of friction angles considered (between 34 and 44 degrees).

Using the same method of determining the shear strength profile of the subsoil and the same method of stability analysis (BTENSAR), it was calculated that the inclusion of the reinforcement caused an improvement in failure heights of 34 per

cent and 17.5 per cent, for the strip and geocell reinforced embankments, respectively (Table 4). Details of the observed and calculated critical slip surfaces, as well as the shear strength profiles adopted in the analyses, are summarized in Figure 11.

	ϕ' Fill (degrees)	Calculated Failure Height for Equivalent Unreinforced Fill (m)	Observed Failure Height (m)	Improvement (per cent)	Calculated Force in Reinforcement at Failure (kN/m)
1984 Strip Reinforced	34	4.6	6.2	34	123
1984 Geocell Reinforced	34	4.0	4.7	17.5	64

Table 4 Calculated Improvement in Failure Height and Force in Reinforcement at Failure

6.2 STRAIN RELATED CAPACITY OF GEOGRID

In general the load and strain mobilized in the reinforcement (Tensar SR-2) were relatively small during construction and up to failure. The maximum load measured in the geogrid under the crest of the strip reinforced embankment was 9.1 and 6.4 kN/strip (1 m wide) at the lower and upper layers of reinforcement, respectively. The maximum load measured prior to failure at the centreline of the geocell reinforced embankment was 19 kN/strip; the corresponding average strain mobilized as indicated by Bison strain gauges was 3.4 per cent. At the crest a maximum load of 8 kN/strip and an average strain of 1.8 per cent were measured at the same time. These strain measurements are in good agreement with

the results of the horizontal extensometers located in the fill close to the reinforcement.

Given the load and strain measured at the same relative location with respect to the centreline, in the geocell reinforced embankment, a load-strain relationship for the reinforcement under field conditions can be plotted as shown in Figure 12. From this the overall stiffness of the transverse member, up to a strain of 3.4 per cent, was 5.6 kN/per cent strain/strip. At these relatively low strain levels the geogrid in both reinforced embankments was in a stable state just prior to failure.

The strength of polymer grids decreases with decreasing strain rates. For the Tensar SR-2 geogrid the minimum tensile capacity (corresponding to strain rates less than 10^{-3} per cent/min) is 45 kN/strip. Higher strain rates give greater capacity and it should be emphasized that with the relatively rapid straining just prior to failure, the rupture load of the geogrid would exceed 45 kN/strip and probably approach 60 kN/strip.

To increase from the low loads measured in the reinforcement just before failure, to some value in excess of 45 kN/strip in the short period between the last reading being taken and the onset of failure, a rapid increase in load must have taken place in the reinforcement and, since this was ineffective in resisting the failure, it is concluded that there was a significant reduction in the strength of the foundation in the period immediately preceding failure.

Given that the estimated capacity of the geogrid of 45 kN/strip represents a "lower bound" for the capacity of the reinforcement, the calculated force in the reinforcement of 123 kN/strip (1 m wide) in the strip reinforced embankment represents a reasonable value for 2 layers of geogrid. The calculated value of 64 kN/m-width of the geocell mattress corresponds to a force in the transverse strip of 50 kN/strip and this is also considered to be in reasonable agreement with the rupture load.

7.0 CONCLUSIONS

From the results of the study the following observations and conclusions can be stated.

- No reliable prediction of the failure height of an embankment on soft clay can be made until reliable data on the strength mobilized in the crust is obtained and the applicability of these data to stability analyses is demonstrated.

- The limit equilibrium analyses described in this report made use of the 4 known test embankment failures on the same site in order to calibrate a method of analysis and interpret the strength data available for the foundation. There is therefore a good degree of reliability for the results.

- The results of the analyses indicate that the strip reinforcement allowed a 34 per cent increase in the failure height, while the geocell reinforcement allowed the embankment to be built 17.5 per cent higher than its "unreinforced" failure height.

- For the conditions of the test embankments it is concluded that the geocell mattress is inferior to the simple horizontal strip layout which is far easier to place and construct.

- The simple method of limit equilibrium stability analysis presented in this report can be used to predict the failure height of a reinforced embankment on a soft clay, provided that the strength of the crust of the clay is known.

- The tensile load at which the Tensar SR-2 failed in the reinforced embankments was at least 45 kN/m, which is consistent with the values back-calculated by limit equilibrium analyses.

- For embankments on soft clay foundations the forces in the reinforcement are a direct consequence of the strains induced in the foundation. The maximum strain in the test embankments was about 3.5 per cent up to and immediately before failure. Creep failure of the reinforcement would not be induced at a strain of 3.5 per cent.

- Relatively low loads were observed in the reinforcement a few hours before failure, and these would have to have increased significantly for failure of the reinforcement to occur. It therefore appears that at failure, the resistance of the subsoil was significantly less than that available a few hours earlier. This confirms a strain softening effect in the foundation during failure as indicated by laboratory tests.

- Comparison of the pore pressure response and the lateral deformations under the embankment suggest that the reinforcement does little to restrain the foundation. Instead, its effect is to hold the embankment together and allow more extensive propagation of failure and consequential deformations in the foundation soil before overall failure is initiated. While reinforcement permits higher embankments to be constructed a correspondingly greater deformation occurs.

- At the present state of knowledge, simple limit equilibrium analysis with realistic properties for the reinforcing elements and the strength of the crust, at its most critical location, offers the most reliable method of design.

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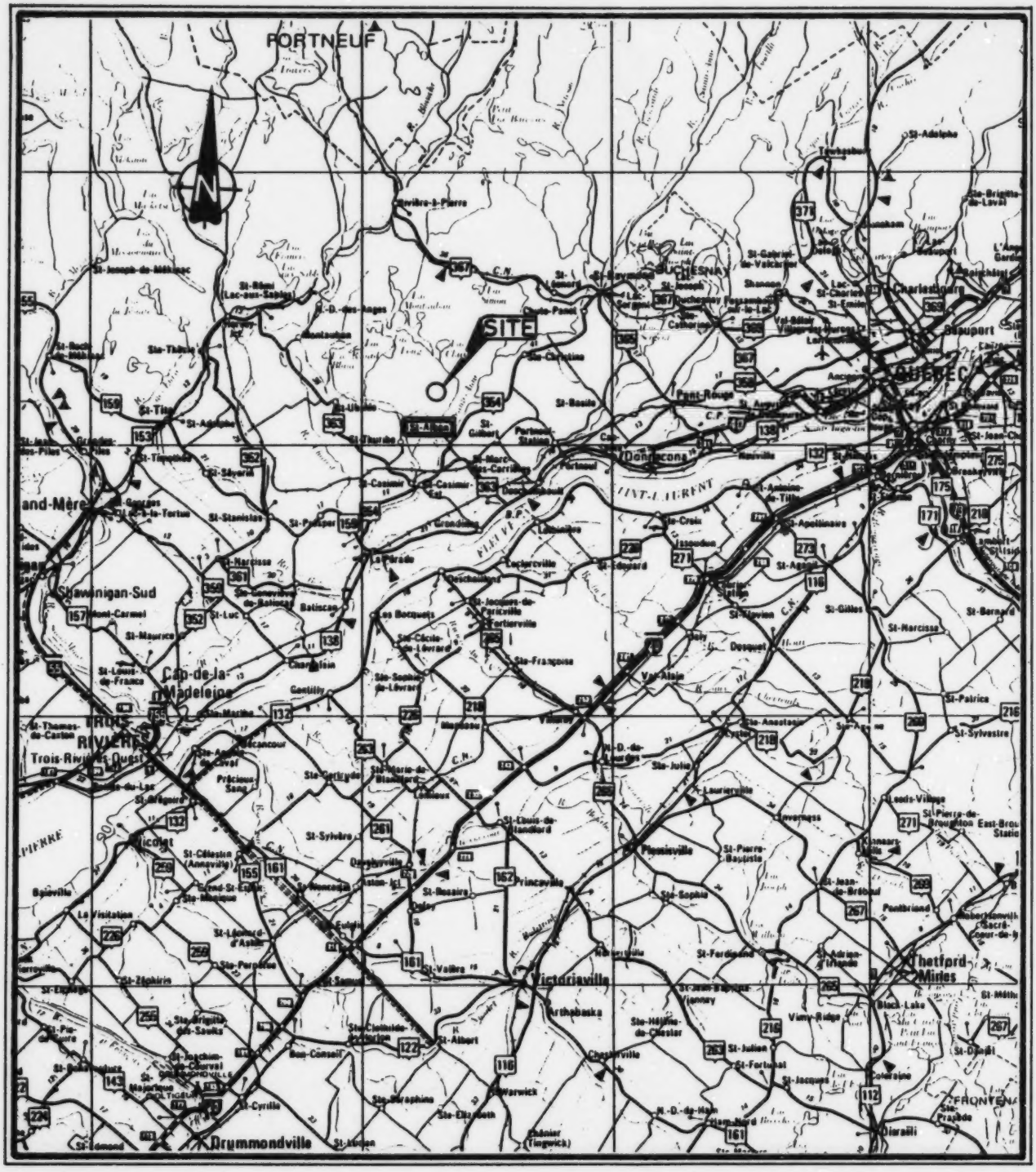
LADD, C.C., and FOOTT, R. (1974). New design procedure for stability of soft clays, ASCE Journal of the Geotechnical Engineering Division, 100(GT7) pp. 763-786.

TRAK, B., LA ROCHELLE, P., TAVENAS, F., LEROUEIL, S., Roy, M. (1980). "A New approach to the Stability Analysis of Embankments on Sensitive Clays", Canadian Geotechnical Journal Vol. 17, No. 4, pp 526-544.

MILLIGAN, V., and LA ROCHELLE, P.R. (1984). "Design Embankments over Weak Soils". Symposium on Polymer Grid Reinforcement in Civil Engineering, ICE, London.

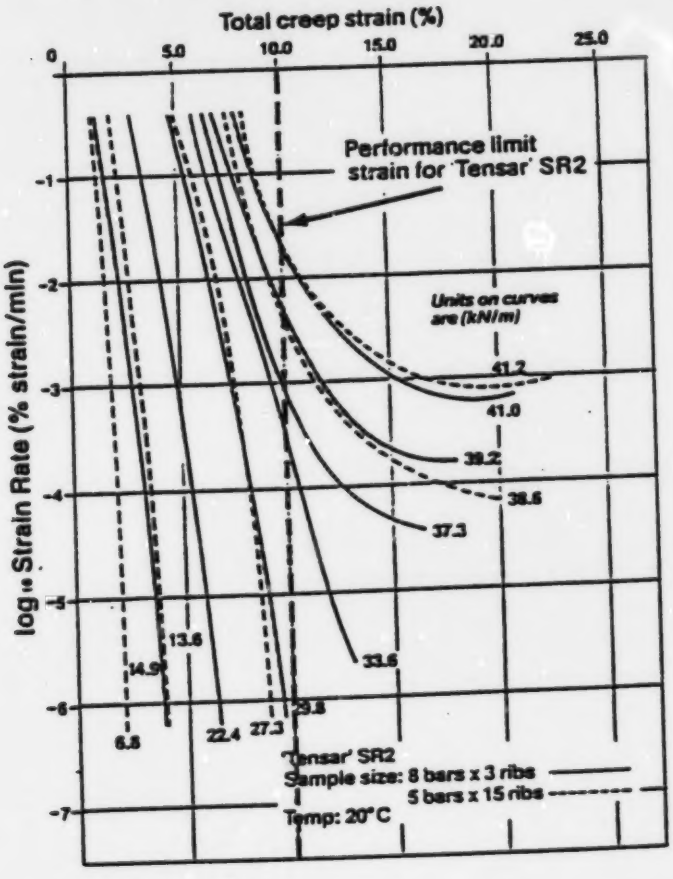
FIGURES



SCALE 1:750

FIGURE 1 SITE LOCATION PLAN

a) CREEP STRAIN RATE VS STRAIN



b) ISOCHRONOUS LOAD STRAIN CURVES

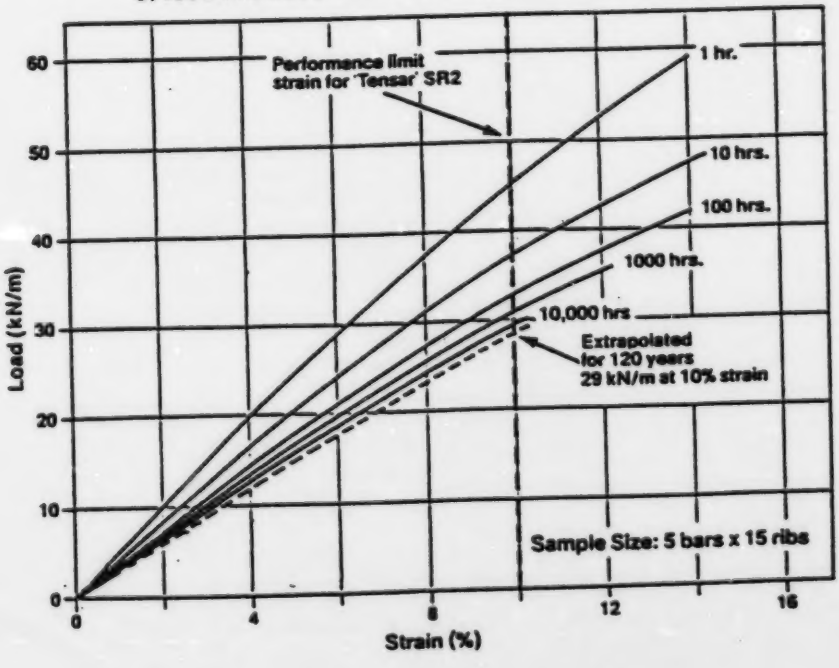


FIGURE 2 TYPICAL PROPERTIES OF TENSAR SR-2

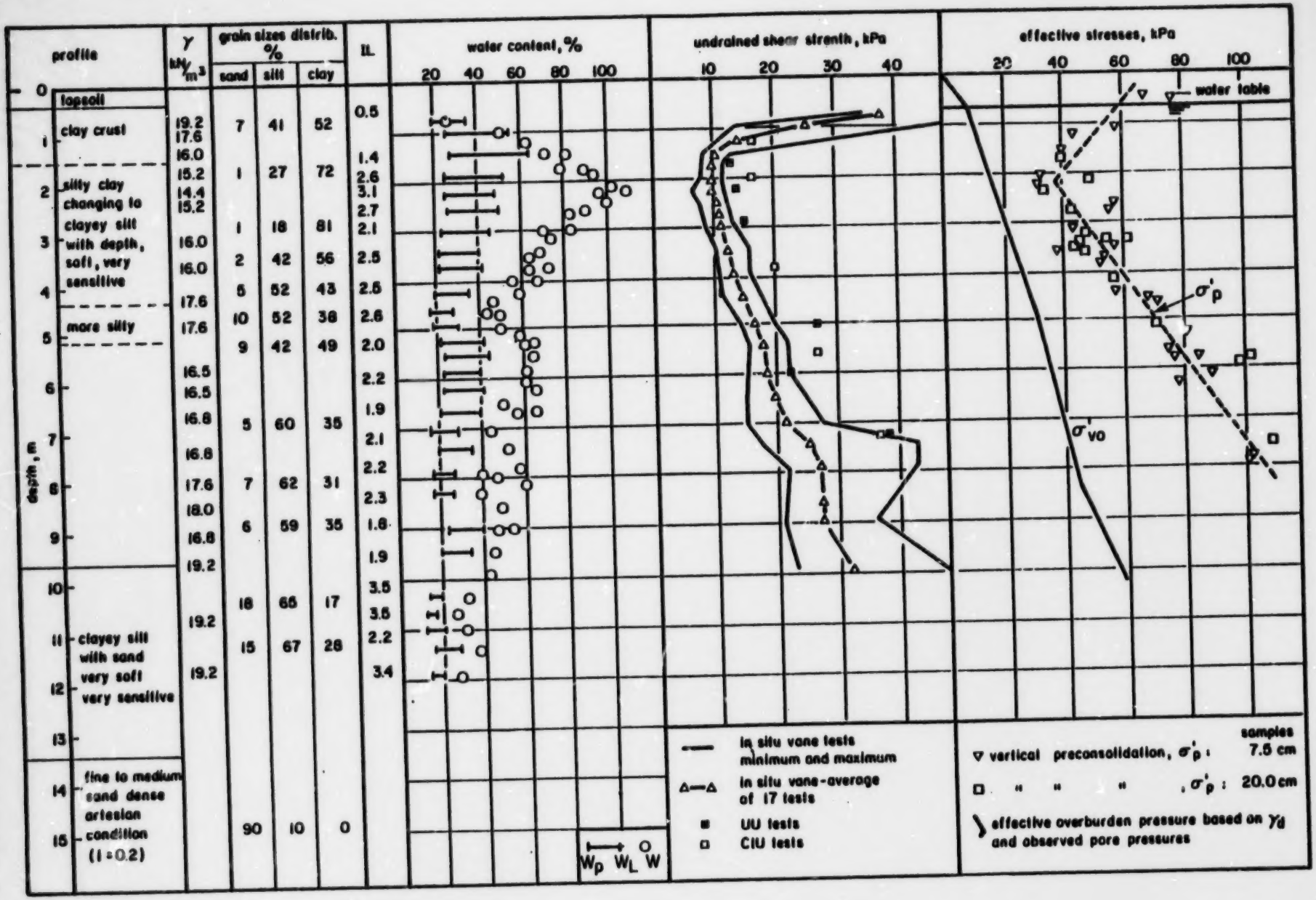


FIGURE 3 TYPICAL SOIL PROPERTIES AT ST. ALBAN TEST SITE

Golder Associates - Université Laval

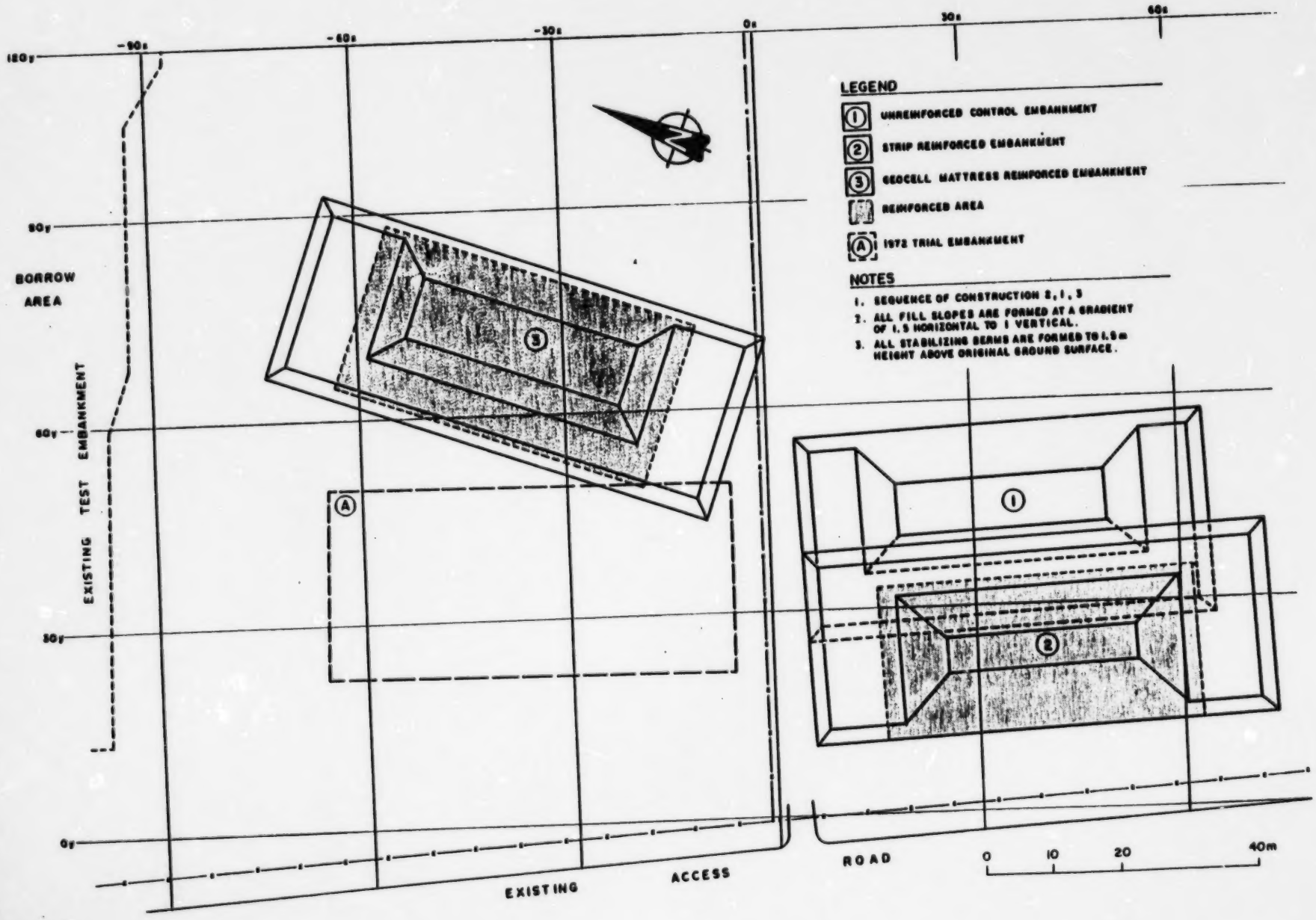
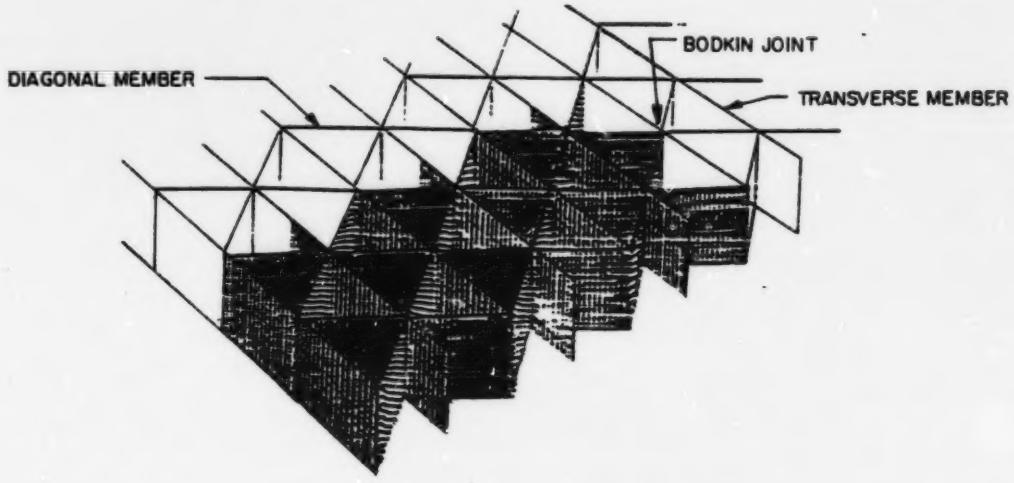
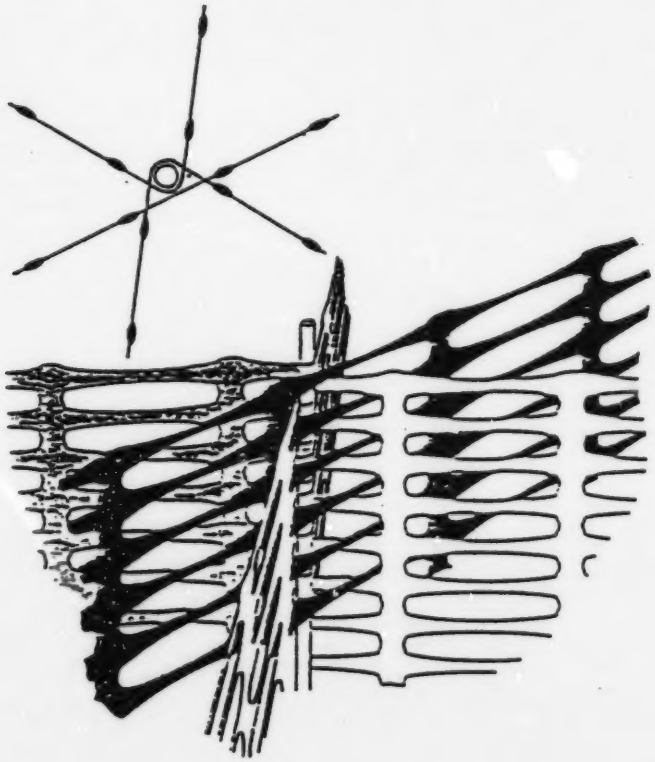


FIGURE 4 GENERAL LAYOUT PLAN

Golder Associates - University Level

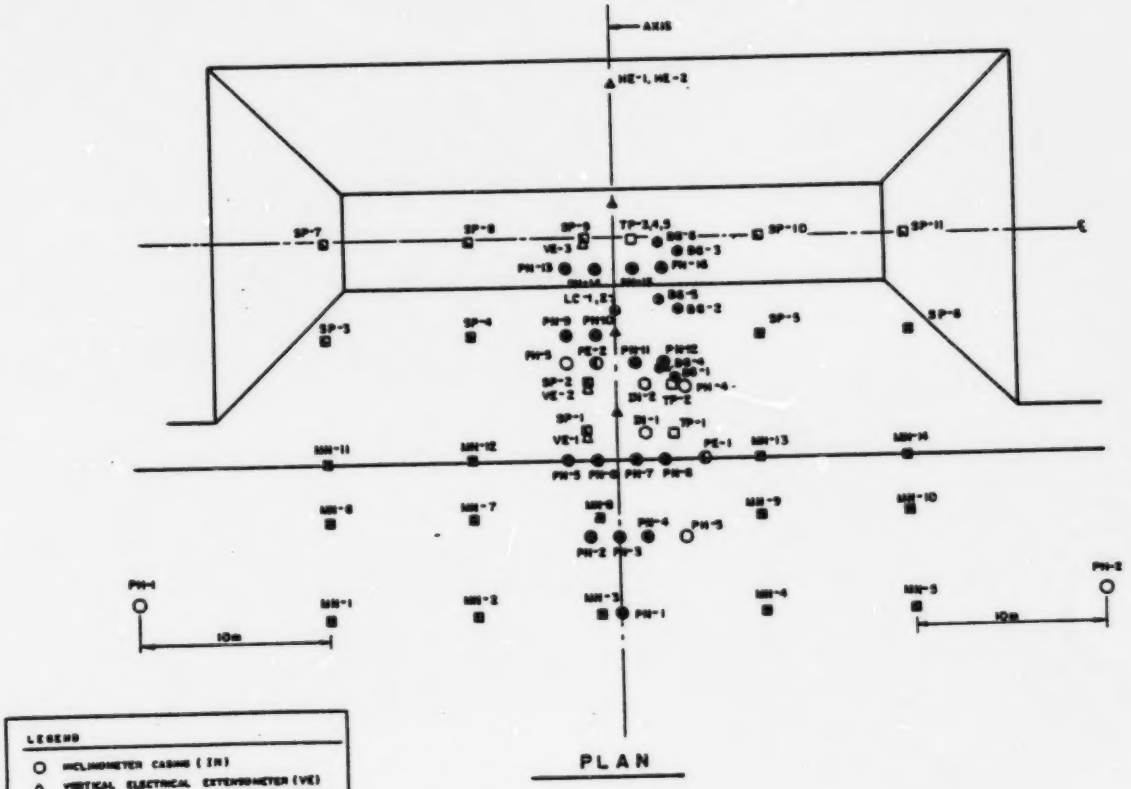


a) ASSEMBLED GEOCELL MATTRESS



b) BODKIN JOINT

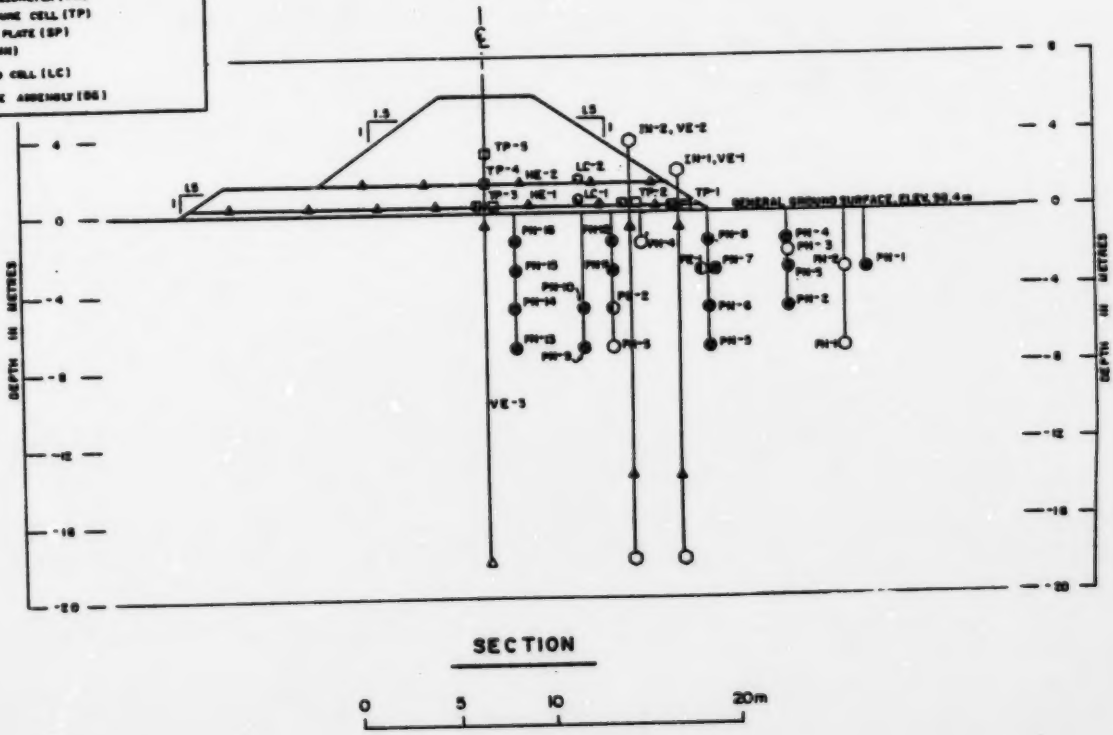
FIGURE 5 PICTORIALS OF GEOCELL MATTRESS AND BODKIN JOINT



PLAN

LEGEND

- INCLINOMETER CASING (IN)
- VERTICAL ELECTRICAL EXTENSOMETER (VE)
- VERTICAL MAGNETIC EXTENSOMETER (ME)
- HORIZONTAL ELECTRICAL EXTENSOMETER (HE)
- PNEUMATIC PIEZOMETER (PN)
- ELECTRICAL PIEZOMETER (PE)
- HYDRAULIC PIEZOMETER (PH)
- TOTAL PRESSURE CELL (TP)
- SETTLEMENT PLATE (SP)
- MONUMENT (MH)
- TENSAR LOAD CELL (LC)
- DIGGS GAUGE ASSEMBLY (DG)

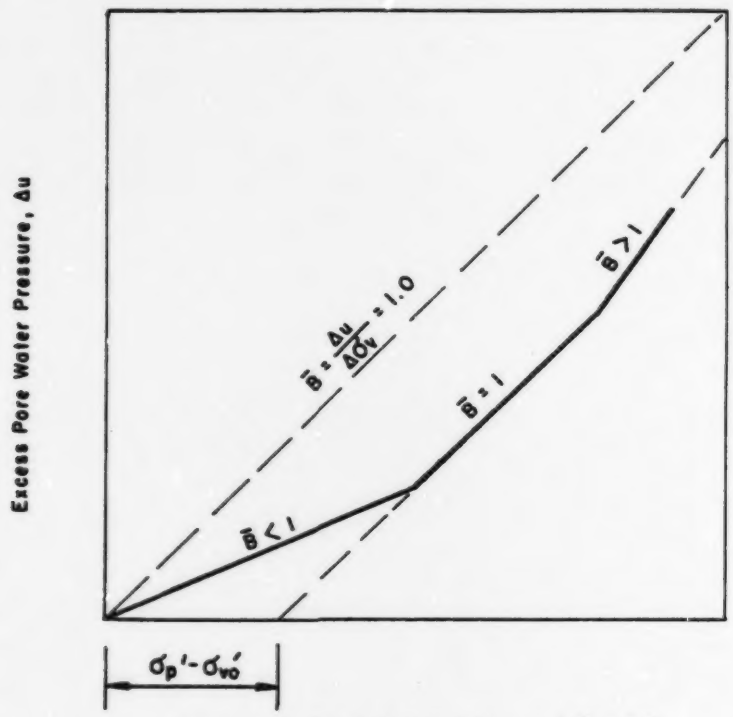


SECTION

0 5 10 20m

FIGURE 6 TYPICAL INSTRUMENTATION LAYOUT (EMBANKMENT NO. 2)

a) TYPICAL PORE PRESSURE RESPONSE UNDER EMBANKMENT CENTRE


Excess Pore Water Pressure, Δu
$\bar{B} = \frac{\Delta u}{\Delta \sigma_v} = 1.0$
$\bar{B} > 1$
$\bar{B} = 1$
$\bar{B} < 1$
$\sigma_p' - \sigma_{vo}'$
Increase in Total Vertical Stress, $\Delta \sigma_v = I_\sigma H$

b) PORE PRESSURE RESPONSE, PN–15

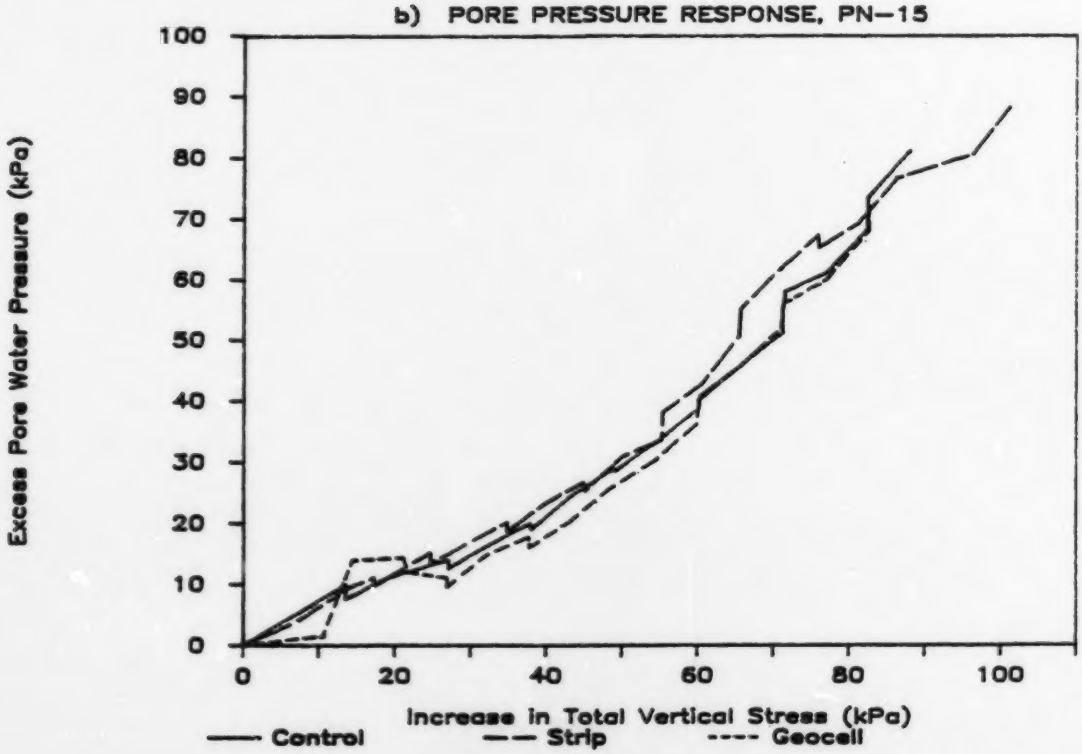
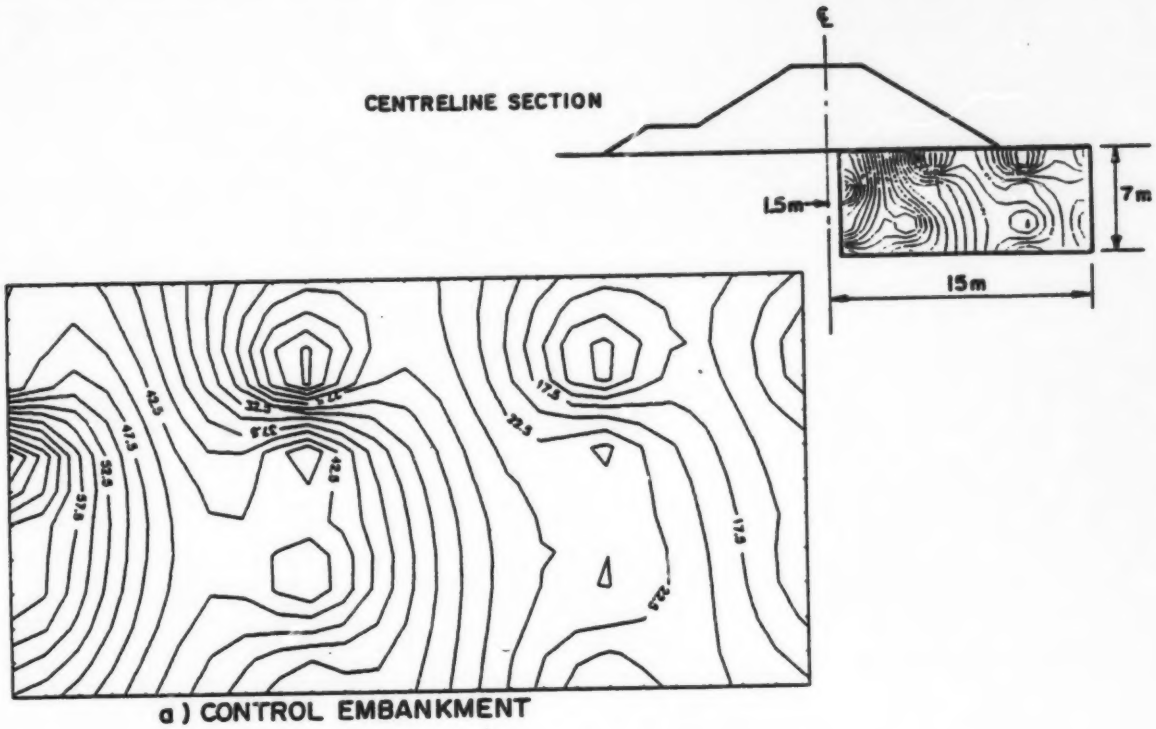


FIGURE 7 VARIATION OF PORE PRESSURE UNDER EMBANKMENT CENTRE



a) CONTROL EMBANKMENT

b) STRIP REINFORCED EMBANKMENT

c) GEOCELL REINFORCED EMBANKMENT

FIGURE 8 EXCESS PORE PRESSURE CONTOURS AT SURCHARGE OF 84 kPa

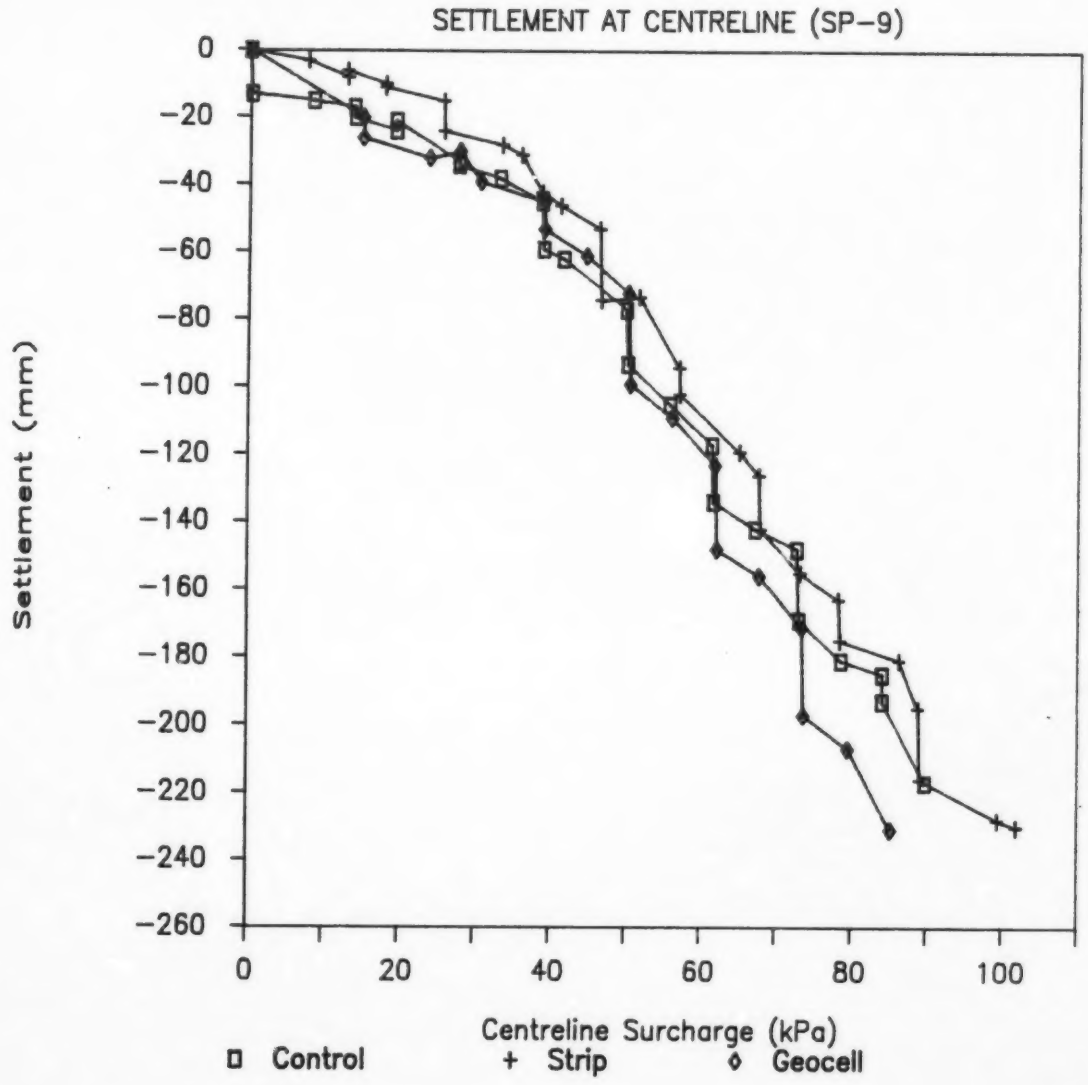
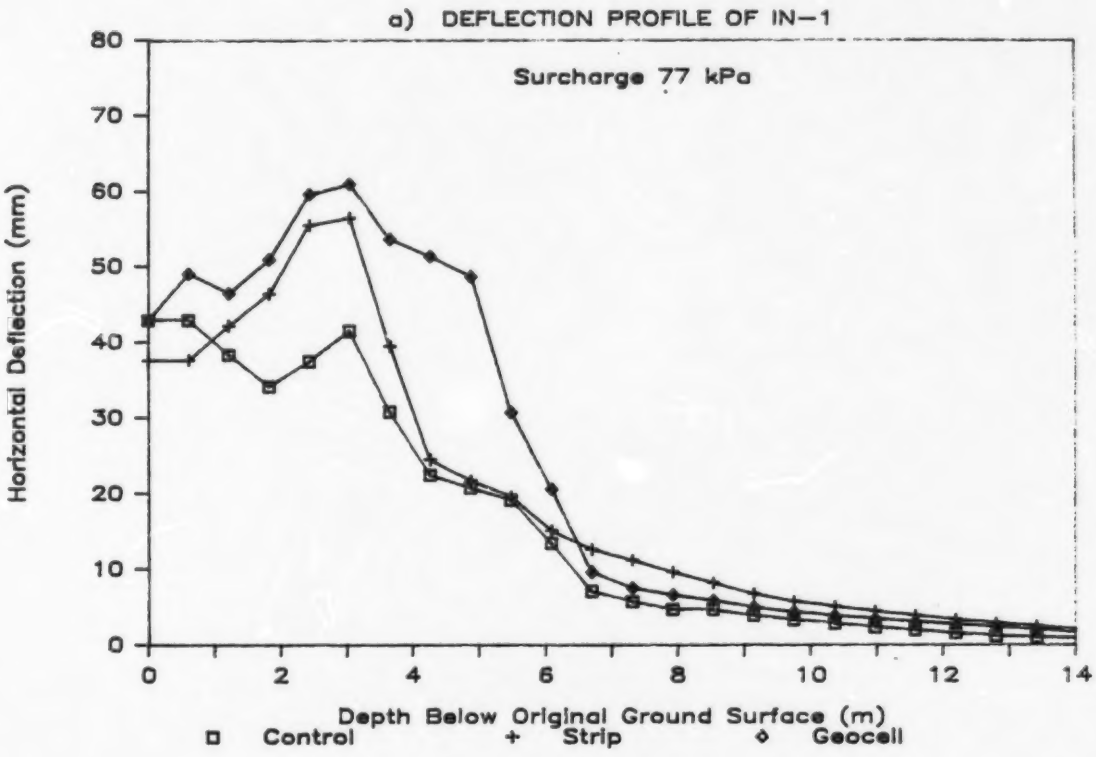


FIGURE 9 COMPARISON OF SETTLEMENTS AT CENTRELINE (SP-9)



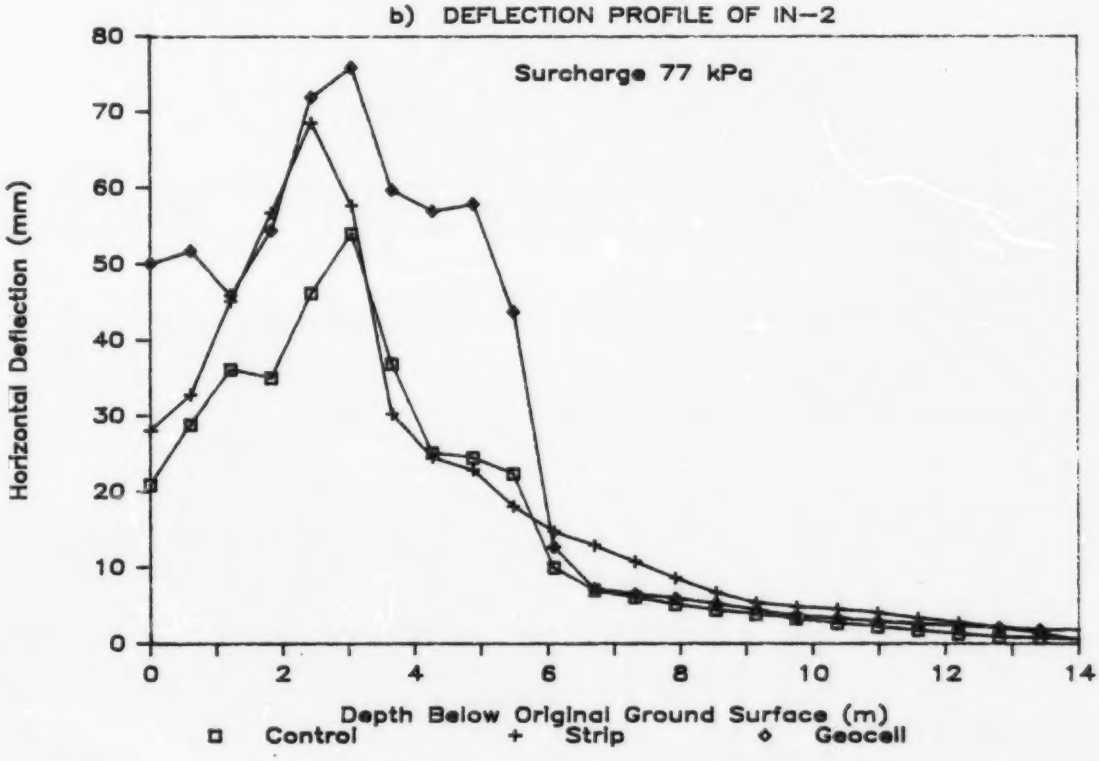


FIGURE 10 COMPARISON OF DEFLECTION PROFILES AT SURCHARGE OF 77 kPa

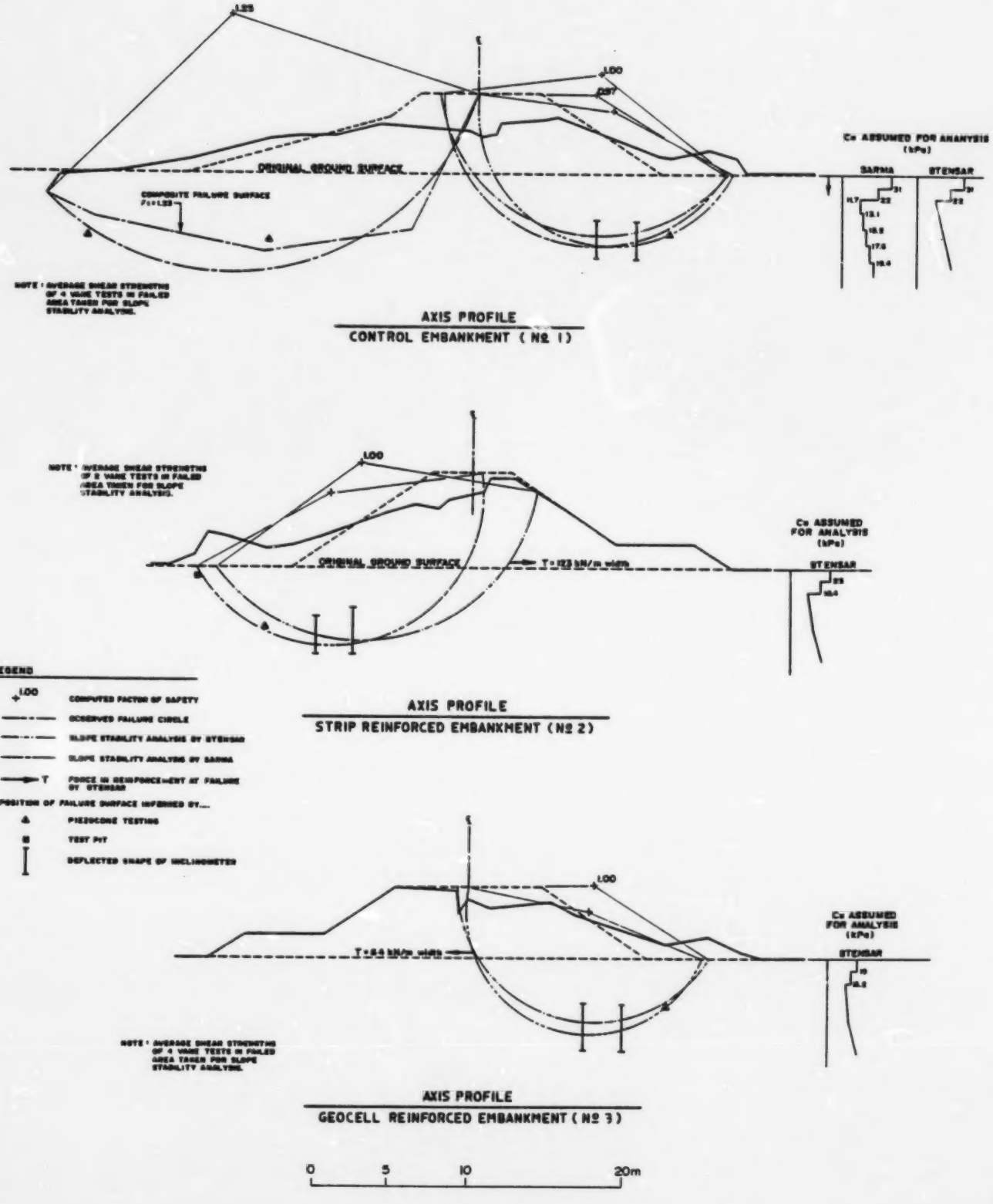

ORIGINAL GROUND SURFACE
COMPOSITE FAILURE SURFACE
Cu ASSUMED FOR ANANYSIS (kPa)
SARMA
STENSAR
NOTE : AVERAGE SHEAR STRENGTHS OF 4 VANE TESTS IN FAILED AREA TAKEN FOR SLOPE STABILITY ANALYSIS.
AXIS PROFILE
CONTROL EMBANKMENT (Nº 1)
NOTE : AVERAGE SHEAR STRENGTHS OF 2 VANE TESTS IN FAILED AREA TAKEN FOR SLOPE STABILITY ANALYSIS.
ORIGINAL GROUND SURFACE
Cu ASSUMED FOR ANALYSIS (kPa)
STENSAR
LEGEND
COMPUTED FACTOR OF SAFETY
OBSERVED FAILURE CIRCLE
SLOPE STABILITY ANALYSIS BY STENSAR
SLOPE STABILITY ANALYSIS BY SARMA
FORCE IN REINFORCEMENT AT FAILURE BY STENSAR
POSITION OF FAILURE SURFACE INFERRED BY...
PIEZOCONE TESTING
TEST PIT
DEFLECTED SHAPE OF INCLINOMETER
AXIS PROFILE
STRIP REINFORCED EMBANKMENT (Nº 2)
Cu ASSUMED FOR ANALYSIS (kPa)
STENSAR
NOTE : AVERAGE SHEAR STRENGTHS OF 4 VANE TESTS IN FAILED AREA TAKEN FOR SLOPE STABILITY ANALYSIS.
AXIS PROFILE
GEOCELL REINFORCED EMBANKMENT (Nº 3)
0
5
10
20m

FIGURE 11 SUMMARY OF STABILITY ANALYSES

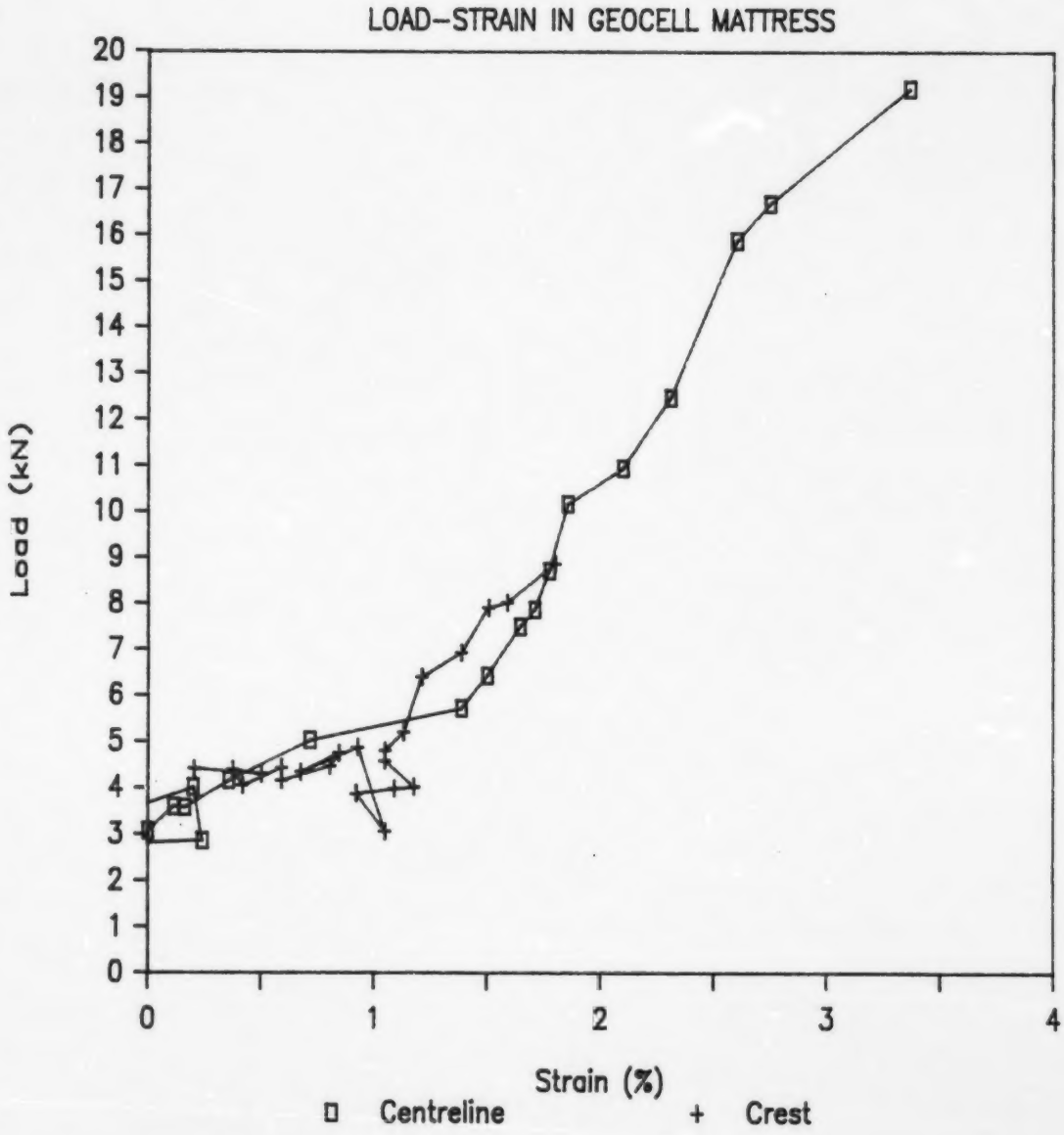


FIGURE 12 LOAD-STRAIN CHARACTERISTICS OF GEOCELL MATTRESS